

Stability of a proposed steepened beach

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Abstract

Existing topography at an operational lateritic nickel facility is such that if the tailings beach slope could be increased to 1-2% for approximately one year, significant costs related to wall building could be deferred. As additional thickening capacity would require significant capital expenditure, polymer treatment technologies have instead been investigated.

As part of the trial process for polymer treatment, the stability of the potential steepened beach was assessed. While beach slopes of the range targeted here would, for many materials, be unlikely to produce an unstable landform, lateritic nickel tailings develop very low dry densities, consolidate slowly, and contain hypersaline pore fluid. This combination results in a lower resistance to slope instability, all else being equal. Further, polymer treatment technologies, while providing benefits through rapid dewatering and beach slope development, have been shown to result in an increased brittleness for some materials.

Laboratory characterisation of the polymer treated material was first undertaken, by means of (i) monotonic, cyclic and post-cyclic simple shear testing, to assess the potential for strength loss of the material, (ii) laboratory-scale shear vane testing, to estimate residual shear strength, and, (iii) settling and consolidation tests, to provide input to consolidation modelling.

A consolidation model of the proposed steepened beach was then developed. Outputs from this model were used as the basis for infinite slope stability analyses on a variety of scenarios. It was found that over the one year planned deposition period, beach slopes of 1.5% or less are likely to result in a satisfactory factor of safety (FS) for the steepened beach.

1 Introduction

The existing topography at an operational lateritic nickel tailings facility is such that if the beach slope of the material were to be increased to 1-2% from the current typical slope of 0.7%, significant deferral of wall raising activities and related capital expenditure could be realised. Deposition in such a manner could be conducted for approximately one year, on the basis of current production rates and site geometry, with an expected rate of rise (ROR) of 5 m/year, were this undertaken. As this situation is temporary owing to current site conditions, capital outlays for additional thickening capacity to produce an increase in beach slope would not be a prudent investment. However, the implementation of polymer treated (PT) tailings at or near the point of deposition offers the potential to increase the beach angle of the tailings with significantly lower capital costs than through additional thickening.

As part of the assessment of the potential for the steepened beach concept, laboratory testing was undertaken to characterise the behaviour of the PT tailings. The results of this testing were used in consolidation and stability analyses, to assess the expected stability of the proposed steepened beach.

2 Previous relevant studies

2.1 Steepened beach stability

A number of previous studies have been undertaken to assess the expected stability of proposed unconstrained tailings beaches, most of which were developed from thickened tailings deposition. Each of

these studies cited below was undertaken on relatively small quantities of material available in the design stage. These are summarised as follows.

The first published study to assess the stability of a steepened beach concept appears to be that of Poulos et al. (1985), who investigated the stability of a proposed alumina residue tailings. They found that if sufficient straining of the tailings were to occur such that the strength of the material was reduced to residual conditions, it was unlikely the beach would be stable with a beach slope of 5%. However, cyclic assessment of the tailings indicated that sufficient strains were unlikely to be generated in the design seismic event.

Seddon et al. (1999) assessed the stability of a planned down-valley zinc tailings beach, with an expected slope of 0.5%. A residual strength ratio, i.e. ratio of residual shear stress to vertical effective stress, of 0.04 was obtained through vane shear testing. Post-seismic stability analyses indicated that in the early stages of the facility development, minor flow slides were possible should seismic loading develop residual strengths in the material. However, as deposition continued the potential for flow liquefaction would be eliminated by means of evaporative drying effects and increased consolidation of the material. The starter embankment for the facility was sized to accommodate potential flow slide events in the early stages of deposition.

McPhail et al. (2004) assessed the expected in situ state of a proposed copper-gold tailings thickened deposit, through comparison of the consolidation densities to the critical state line of the material. This indicated that the material was likely to be denser than the critical state in situ, and hence exhibit dilative behaviour. Recent interpretation (Reid & Fourie 2015) of in situ testing undertaken within the material suggests that the realised in situ states are somewhat denser than predicted from the laboratory testing, potentially resulting from air drying effects.

Seddon (2007) obtained a residual strength ratio of 0.05 by means of vane shear testing within a portion of copper tailings undergoing air drying. Post-seismic stability analyses across a range of beach slopes indicated that, assuming the phreatic surface was at the level of the failure surface, beach slopes of up to 5% would be stable.

Li et al. (2009) outlined methods to develop residual strengths based on material state with respect to the critical state line (CSL), and utilise these within a finite-strain consolidation model to assess post-seismic stability throughout the planned deposition process. This indicated the sensitivity of rate of rise and in situ state to the resulting FS.

2.2 Lateritic tailings

Lateritic tailings often exhibit relatively slow rates of consolidation, and low final dry densities at a given stress when compared to other tailings (Mundle & Chapman 2010; Azam 2011, Locke & Herza 2012). Such behaviour means that at a given deposition rate, the increase of vertical effective stress with depth can be significantly lower than a coarser, denser tailings. This is of importance to the current study as the increase in vertical effective stress with depth directly controls the available shear strength. A lower shear strength would result in a lower FS for a given beach profile.

2.3 Hypersaline pore fluid

Hypersaline pore fluid is frequently used in the processing of ores in Western Australia (Fujiyasu & Fahey 2000; Newson & Fahey 2003; Williams & Seddon 2004; Mundle & Chapman 2010; Mundle et al. 2012). Pore fluid with significant dissolved solids such as salt requires some additional considerations when undertaking the testing and analyses outlined herein.

Laboratory tests must be corrected for salt content, otherwise dry density estimates will be erroneous. These errors will be non-conservative with respect to the calculation of in situ vertical effective stress. The analyses undertaken, in particular the infinite slope stability analysis, must include the pore fluid density in estimates of total stress and pore pressures. While the increased pore fluid density will increase both total

stresses and pore pressures (all else being equal), the increase to pore pressures is proportionately greater, thus resulting in a reduction in vertical effective stress.

2.4 Geotechnical effects of polymer treatment

Polymer treatment of tailings to increase the rate of dewatering, increase shear strength, reduce segregation, and promote a steeper beach angle is an emerging technology of increasing popularity (Dymond 2003; Kaiser et al. 2006; Adkins et al. 2007; Brumby et al. 2008; Vietti et al. 2008; Daubermann & Földvári 2008, 2009; da Silva 2011; Wells et al. 2011). While the implementation of PT has been shown often to result in improved behaviour as desired for the proposed steepened beach concept outlined herein, it has also been shown to affect the geotechnical behaviour of tailings. In particular, PT has been shown, for some materials, to increase the rate of consolidation (Jeeravipoolvarn et al. 2009, 2010; Reid & Fourie 2014), while also potentially increasing the resulting material's brittleness (Beier et al. 2013, Reid & Fourie 2014). This increased sensitivity may partially result from lower densities that sometimes result from PT (Jeeravipoolvarn et al. 2009, 2010; Reid & Fourie 2012; Yao 2012; Reid & Fourie 2014). Alternatively, higher densities have been shown for some lateritic tailings on the basis of PT (Azam 2011).

The available literature therefore suggests that the geotechnical effects of PT should be assessed as part of their use to develop a steepened beach. In particular, the potential for PT to result in a more brittle material must be assessed such that the resulting landform is stable.

3 Laboratory characterisation

3.1 General

The following stages of laboratory characterisation were undertaken, to assess the stability and expected densities of a steepened beach developed by means of PT:

- Polymer screening tests, to assess the optimum polymer and dosage to promote rapid dewatering and strength gain in the tailings.
- Monotonic, cyclic, and post-cyclic simple shear testing of the PT material, to assess the peak undrained strength and potential for cyclic strains to result in a reduction in strength.
- Laboratory-scale shear vane testing, to establish the residual strength of the PT material under high strains.
- An undrained settling test, to establish initial settling rate and an estimate of settled dry density.
- Slurry consolidometer testing, to establish the increase of dry density with vertical effective stress, and the permeability - density profile for the PT material.

The lateritic tailings consist of 87% fines (<0.075 mm) by mass, and have a liquid limit of 55 and a plasticity index of 7. The liquid limit and plasticity index indicated are not corrected for salt.

The majority of the dissolved solids within the tailings pore fluid consist of magnesium sulphate. On this basis, there was uncertainty as to whether the proposed relationship between pore fluid density and dissolved solids for sodium chloride salts (McCutcheon et al. 1993) would be valid. Therefore, density was measured for a sample of the pore fluid, and the dissolved solids measured through oven drying. This indicated a fluid density of 1.17 g/cm³, and a dissolved solids concentration of 247 g of solids per kg of water. These relationships were used to correct for salts in all the laboratory testing outlined below. They are also very similar to those observed when the dissolved solids are made up of sodium chloride.

3.2 Polymer treatment

Polymer screening was undertaken by BASF Australia Ltd, to obtain a product and dosage that resulted in rapid dewatering and an increase in yield stress of the material. A dosage rate of 160 g dry grams of polymer per dry tonne of tailings, added to the slurry at a 0.025% by mass concentration, represents the

current laboratory optimum. However, it is expected that this may be refined as field trials commence. Polymer solution was added to the tailings as they were being mixed within a Hobart mixer. Mixing was undertaken for approximately 10 seconds following polymer addition. Samples were prepared in batches as necessary for testing. Polymer treated material exhibited consistent behaviour across different batches, as outlined below.

Early polymer dosage trials were performed on slurry at 30% solids by mass, consistent with the typical deposition slurry density. However, further refinements indicated that dilution of the solids to 25% solids by mass may result in improved behaviour. As the design polymer treatment technique was evolving during the laboratory study, all the tests were not undertaken on material treated in the same manner. The laboratory scale shear vane testing was performed on material dosed at 30% solids, while the remaining tests were dosed at 25% solids. The potential for the slightly different polymer treatment techniques to influence the results obtained is acknowledged. Rather than undertake additional laboratory testing, the intention is to perform additional in situ and laboratory testing on sample produced from field-scale polymer treatment.

3.3 Simple shear

Testing was undertaken on 63.8 mm diameter samples. Material was extruded from one dimensional consolidation tubes into a ring and trimmed to the required length. Sample height during shear/cycling stages (i.e., after consolidation) averaged 19 mm. Tests were undertaken under constant volume conditions (Finn 1985; Dyvik et al. 1987), which were maintained through active computer control. Samples were laterally restrained by a stack of Teflon rings. Monotonic and post-cyclic tests were sheared at approximately 5% strain per hour, while cyclic tests were tested at a frequency of 0.1 Hz. Cyclic tests were taken to different levels of strain, to assess the dependency of strain on the resulting post-cyclic strength.

Cyclic testing was undertaken at 100 kPa vertical effective stress. As will be seen, this is a higher vertical effective stress than is expected within the steepened beach concept proposed. However, the control of cyclic tests can be difficult at lower effective stresses. In addition, the influence of membrane and ring friction can then become significant as a proportion of the cyclic loads applied. As vertical effective stress has been shown (Wijewickreme et al. 2005; James et al. 2011), for some tailings, to have a relatively small impact on cyclic resistance ratio (CRR), testing was undertaken primarily at 100 kPa.

The cyclic resistance of PT and untreated samples are presented in Figure 1. The results of the cyclic testing indicate that, for the lateritic nickel tailings tested, PT results in a lower cyclic resistance. It is also noted that the results for the two materials tested in this is similar to those previously published for tailings produced through the processing of lateritic soils, e.g. Wijewickreme et al. (2005).

The average dry density for untreated samples were slightly higher than PT samples (0.98 versus 0.94 t/m³), which may be contributing to the lower cyclic strength following PT. However, it is noted that estimating density in the simple shear device for materials with hypersaline pore fluid is likely of limited accuracy. For soils with water as pore fluid, additional water can be used to flush all of the solids from the test into a tin, then dried to obtain mass of solids. However, with hypersaline materials, additional water to the sample is undesirable, as the water evaporated from the sample is used to calculate the quantity of dissolved solids in the sample. While density of the untreated material may be slightly higher than PT samples, it is noted that soil fabric (i.e. the arrangement of particles at a microscopic scale), has been shown to significantly influence cyclic strength for samples prepared to the same density (Mulilis et al. 1977; Ladd 1977). It is therefore possible that PT is resulting in a fabric with reduced resistance to cyclic loading.

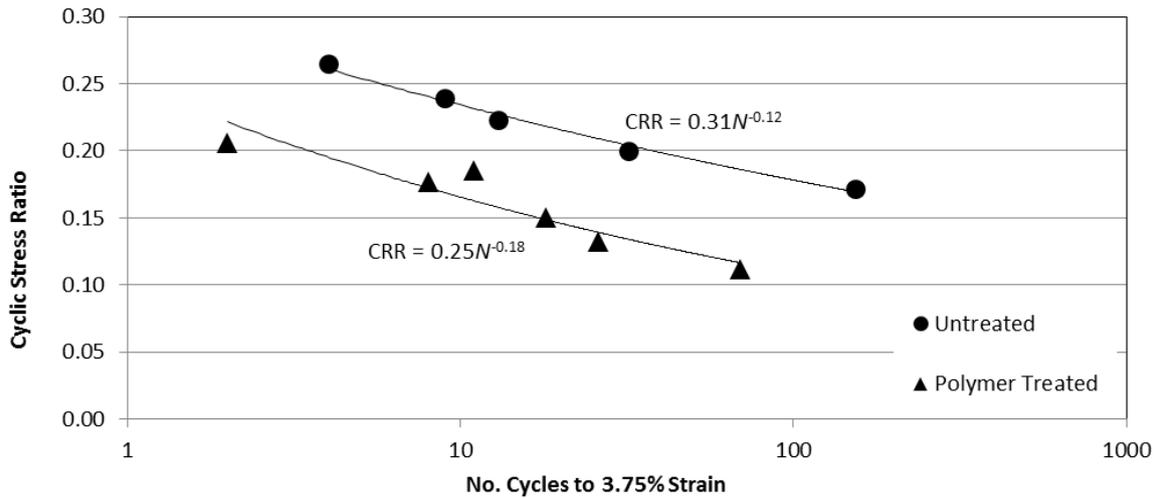


Figure 1 Cyclic test summary

While the development of cyclic strains and pore pressure under sufficient seismic loading is evident for the PT material, it is the post-seismic (post-cyclic) strength after such strains that is likely to govern the stability of the proposed steepened beach. The strengths obtained through post-cyclic monotonic shearing of the samples are presented in Figure 2 as strength ratios of vertical effective stress. The results are plotted in the form proposed by Castro (2003), wherein post-cyclic strength is referenced to the maximum cyclic strain mobilised during cyclic loading. This enables the potential decay in post-cyclic strength, if evident, to be discerned. The influence of the cyclic strain magnitude on post-cyclic strengths of some soils has been noted previously (Thiers & Seed 1969; Yasuhara 1994; Ansal et al. 2001; Castro 2003; Yasuhara et al. 2003; Erken & Ulker 2007; Soroush & Soltani-Jigheh 2009; Friedel & Murray 2010; Dahl et al. 2014; Pillai et al. 2014).

Included in Figure 2 are the peak undrained strengths obtained through monotonic testing, and the average residual strength of PT material from shear vane tests (discussed below). Generally, the PT samples are seen to exhibit lower post-cyclic strengths. Further, while for both untreated and PT materials a decrease in post-cyclic strength is observed with increase cyclic strain, this appears to be more pronounced for PT material. The trend produced also suggests that at cyclic strains above 20-25% the post-cyclic strength of PT material may approach the residual strength obtained from shear vane testing. The estimates of residual shear vane strength are discussed in the following section.

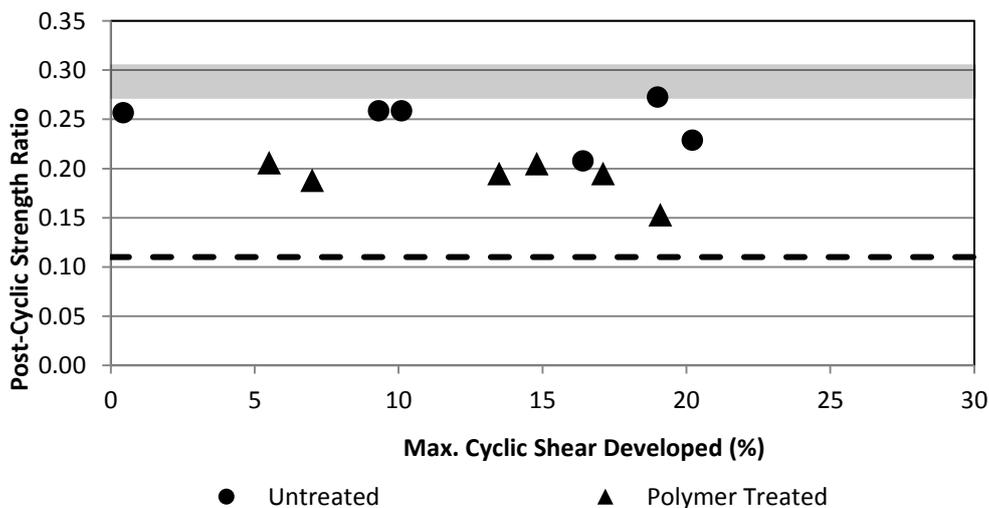


Figure 2 Post-cyclic test summary

3.4 Laboratory-scale shear vane

As indicated from the post-cyclic testing undertaken, it seemed likely that strength loss could be induced in the PT material under sufficient cyclic straining. On this basis, the residual strength of the material was assessed by means of shear vane testing in the laboratory. The residual strength obtained with a shear vane is typically considered the ‘worst case’ strength that a soil can mobilise, regardless of the strain magnitude.

Laboratory-scale shear vane testing to assess peak and residual undrained strengths for planned tailings deposits has been presented by Poulos et al. (1985), Robinsky (1999), Seddon et al. (1999), and Seddon (2007). To produce samples for testing across a range of densities and stresses, previous studies have prepared slurry to different densities, utilised a one dimension load system, or undertaken testing on material undergoing air drying.

The authors have adopted the method proposed by Robinsky (1999), wherein material is prepared in a 150 mm diameter column, and testing undertaken while the sample is under a known one dimensional load. The process in this study involved:

1. Polymer treating the sample, as outlined in Section 3.2.
2. Pouring into a column consisting of two 150 mm diameter permeability moulds fixed together, with drainage media on the base.
3. Allowing sample to settle overnight.
4. Placing filter and drainage media on the surface of the settled sample, and applying increasing one dimensional loads.
5. Once the sample had consolidated sufficiently, removing the upper 150 mm diameter mould, and removing the top loading plate.
6. Switching to a top loading plate which included a small hole, of sufficient diameter that the shaft of a shear vane could pass through the hole (but not the vane blades, which are much larger).
7. Inserting the vane into the soil, then placing the top loading plate over the vane shaft.
8. Applying additional loads by means of a triaxial load frame, then undertaking a number of shear vane tests to peak and residual conditions.

The results of the shear vane testing are outlined in Table 1. The average residual strength ratio obtained was 0.11, with no variation seen based on vertical effective stress observed. This result is consistent with the residual strengths measured with a vane in other low plasticity tailings and natural soils (Castro 2003; Stark et al. 2012).

The peak strength ratios measured averaged approximately 0.90. This was an unexpected result, as the peak undrained strengths measured in the simple shear averaged 0.30. A ratio of 0.90 would typically indicate dilation, as this strength is higher than the expected drained strength of the material. The authors have no explanation for this behaviour. However, it is noted that unexpected dilative responses from polymer treated materials with in situ test techniques have been previously noted (Reid & Fourie 2014). Further, the apparent brittleness resulting from the high peak strengths may be consistent with the higher brittleness values observed in field testing of polymer-treated tailings (Beier et al. 2013).

Table 1 Shear vane test results

| Vertical effective stress (kPa) | No. of tests | Average peak undrained strength ratio | Average residual strength ratio |
|---------------------------------|--------------|---------------------------------------|---------------------------------|
| 29 | 4 | 0.96 | 0.11 |
| 50 | 2 | 0.85 | 0.11 |

It is noted that the vertical effective stresses tested are higher than those which will be relevant to the deposition planned (discussed further later in this paper). However, the residual strength ratio is consistent at the two vertical stresses tested. Further, for a predominately fine-grained soil such as the lateritic tailings tested, it is likely that the normal consolidation line and critical state line of the material are parallel. This indicates that the residual strength ratio is likely to be similar across a range of stresses, e.g. Olson and Stark (2003).

3.5 Settling and slurry consolidometer

A batch of tailings at 25% solids were treated with polymer as described above, and immediately poured into a 1 L graduated cylinder and a slurry consolidometer column (Sheeran & Krizek 1971). The depth of material vs. time was monitored in the 1 L graduated cylinder to establish initial settling rate, and hence initial permeability (Takada & Mikasa 1986), and an estimate of settled dry density.

The material poured into the slurry consolidometer was allowed to settle overnight, then the top loading system was assembled, and loading increments applied to the material from 5-100 kPa. Constant head permeability tests were undertaken at the end of the slurry consolidometer load stages from 10-100 kPa. Tailings pore fluid was used as both the back pressure and constant head test inflow fluid, to prevent the flushing of the salt within the sample during testing. The pore fluid was kept separate to the pump controllers used by means of hazardous water interface systems.

The dry densities estimated from the slurry consolidometer testing are presented in Figure 3, along with the results from settling, simple shear, and shear vane tests. It can be seen that reasonable agreement is apparent between the various test methods, despite their different sample size and preparation histories.

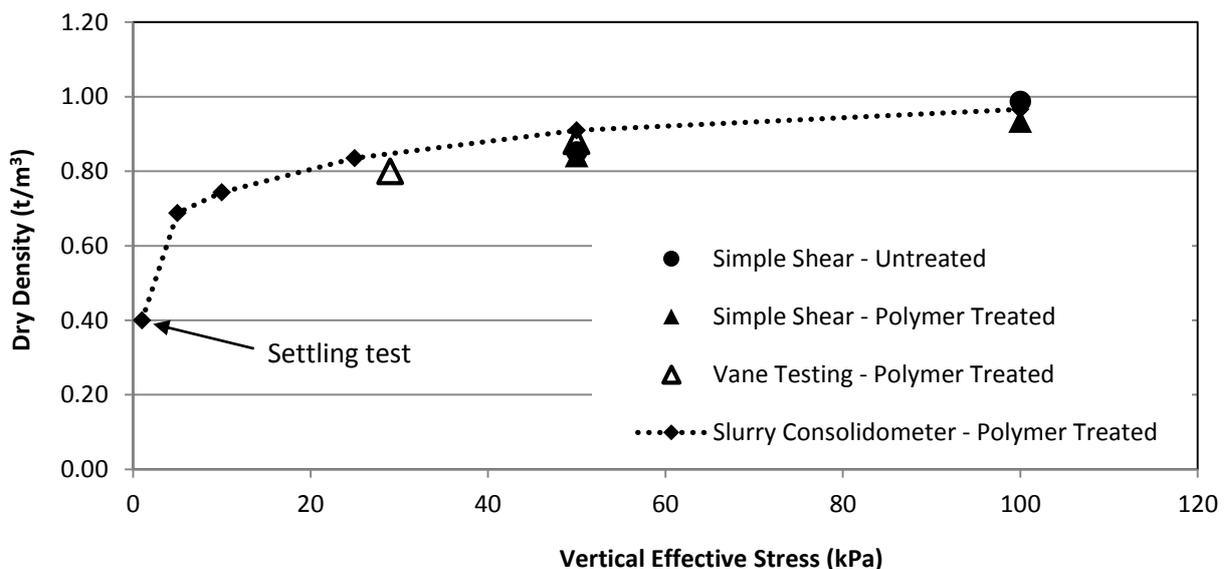


Figure 3 Density results

The permeability results obtained by means of constant head tests in the slurry consolidometer, and as estimated through initial settling rate (Takada & Mikasa 1986) in the undrained settling test are presented in Figure 4. The proposed permeability – void ratio relationship fit to the results, which is used in consolidation modelling (see below), is included.

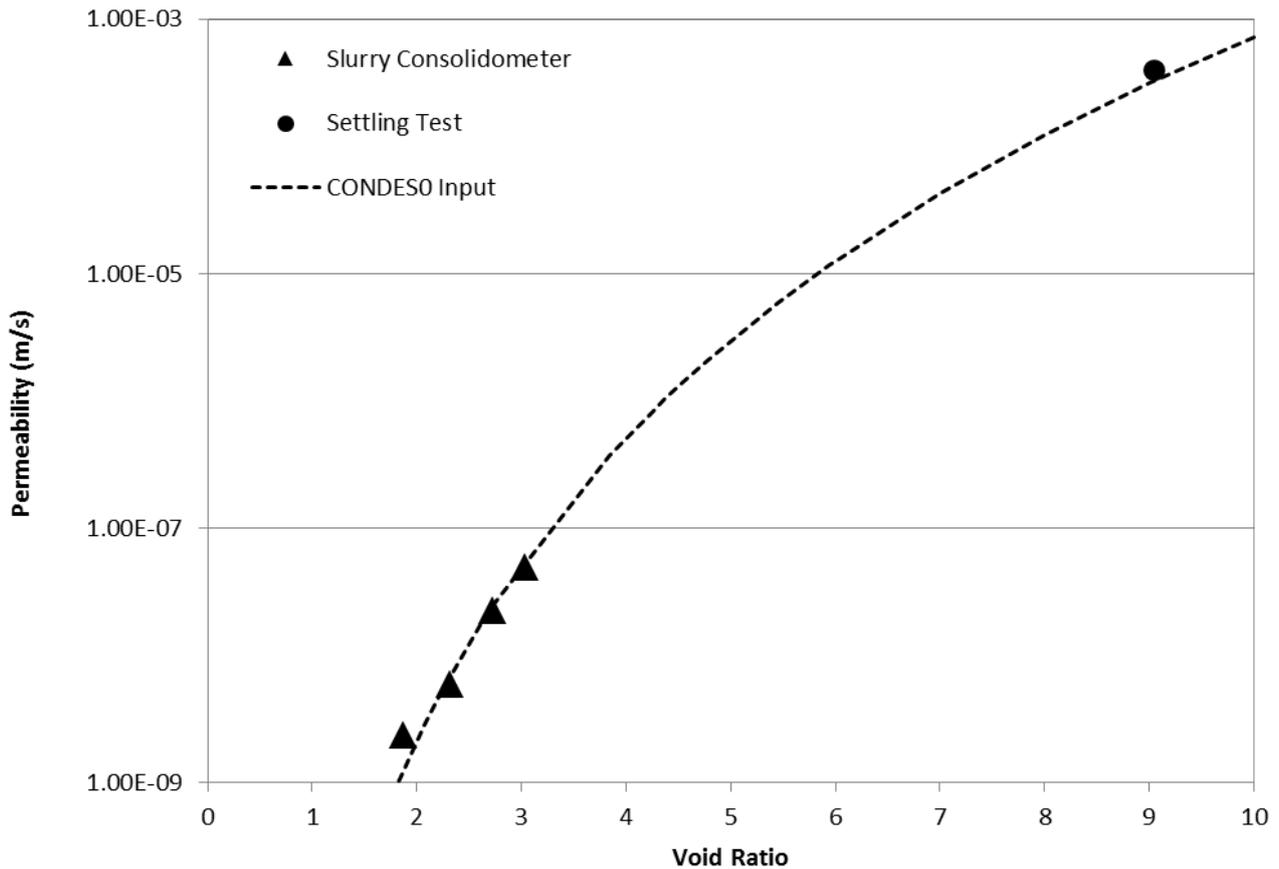


Figure 4 Permeability results

It is noted that the estimation of permeability by means of initial settling rate, and constant head tests from 10 kPa vertical effective stress and higher results in a lack of data points between void ratios of 3 and 9. The authors would, for an untreated material, have considered preparing material at a range of different slurry densities to achieve permeability estimates through settling rate from a range of initial void ratios. However, given the PT nature of the material, this was not practical. While the relationship developed herein is deemed sufficient for current purposes, the authors acknowledge that this study represents a situation where consolidation testing by means of a seepage-induced consolidation test (Abu-Hejleh et al. 1996) would have offered significant advantages.

4 Consolidation and stability analysis

4.1 Consolidation analysis

A one dimensional consolidation model was prepared using the finite-strain code CONDES0 (Yao & Znidarcic 1997). Finite strain models enable many of the salient features of tailings consolidation to be simulated, including incremental deposition, significant variation in material properties with stress, and the relatively large displacements typical of self-weight consolidation. They provide a more robust analysis of self-weight consolidation problems when compared with small-strain analysis techniques, e.g. Schiffman et al. 1984). CONDES0 enables user-input of unit weight of fluid, thus enabling the density of the hypersaline pore fluid to be included in the modelling.

The consolidation model was developed with a rate of rise of 5 m/year. As the input to CONDES0 is the settled depth of tailings per unit time, this input was adjusted until an actual rate of rise of 5 m/year resulted. This was undertaken iteratively with cognisance of the range of dry densities estimated from the model, as this will influence ROR.

It was assumed that the foundation soils were impermeable. The foundation soils relevant to the planned deposition area consist of clayey silts and silty clays. While some dissipation of excess pore pressure into these materials is expected, it is unlikely that they will enable full dissipation of excess pore pressure at the base of the tailings. Therefore, a conservative assumption of an impermeable foundation has been made for the purposes of the analyses. The material parameters for the tailings were developed from the settling and consolidation testing outlined above.

The results of the consolidation analysis at 365 days of deposition are presented in Figure 5. As can be seen, the deposition rate is such that consolidation is not completed during deposition. In general, the dissipation of pore pressure is about 60% complete at 365 days. This degree of consolidation is consistent with analytical expressions for small-strain incremental consolidation (Gibson 1958), based on an average coefficient of consolidation (c_v) of 4 m²/year for the material at the relevant stress states.

The profiles of vertical total and effective stress, and pore pressure, developed in the consolidation modelling are used as inputs to stability analyses, as outlined below.

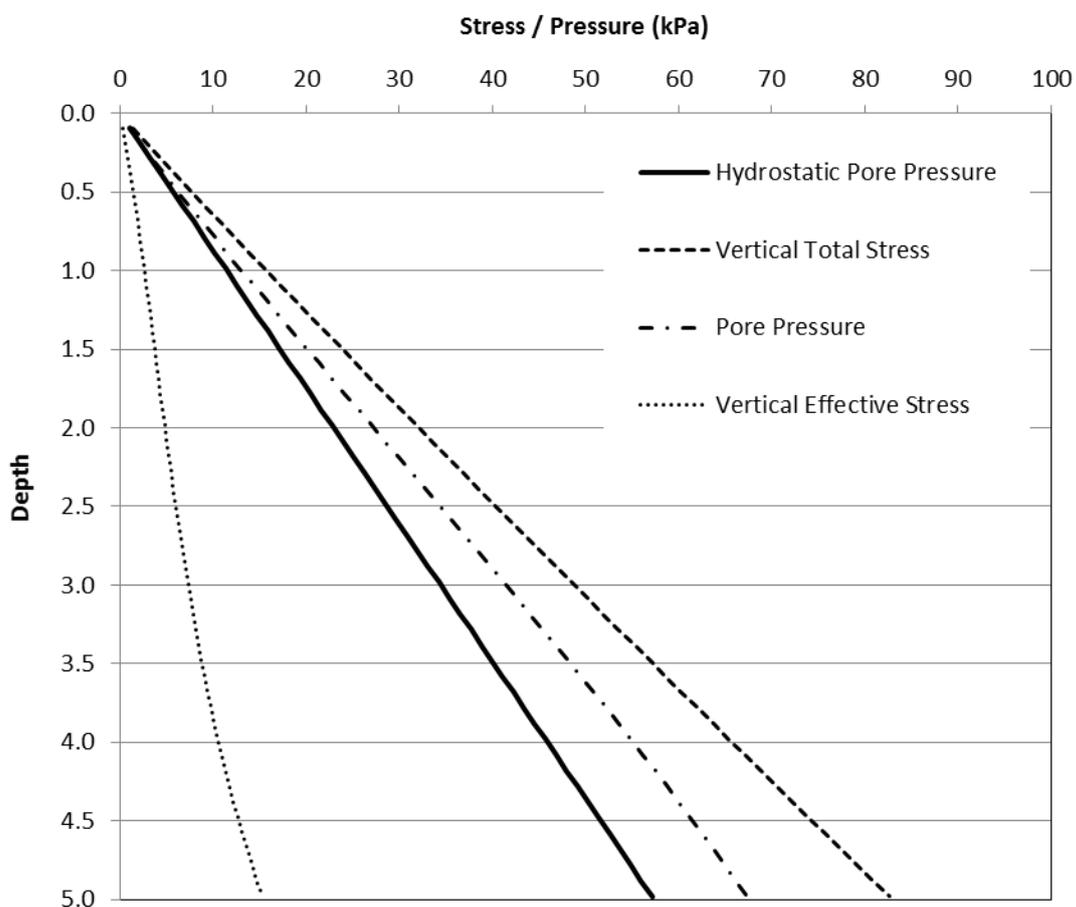


Figure 5 Consolidation analysis – one year of deposition

4.2 Strength loss considerations

Monotonic and post-cyclic simple shear testing of PT material indicated a peak undrained strength ratio of 0.30, with a CRR of 0.15. The 1 in 10,000 average annual exceedance peak ground acceleration (PGA) for the site is 0.225 g. Such a combination of seismic load (and resulting cyclic stress), to cyclic resistance would often, for many tailings, not result in large cyclic strains, if the material was not saturated to the surface. However, with the lateritic tailings under consideration, vertical effective stresses increase with depth very slowly with an assumed ROR of 5 m/year. This results in a significantly lower FS against cyclic liquefaction, and hence expected cyclic strains, under the design earthquake event when analysed by means of the simplified method (Seed & Idriss 1971).

Further to the above, for a mass of deposited material with relatively low vertical effective stress increase with depth, and hypersaline pore fluid of high density, it is unclear if the techniques to estimate cyclic loading with depth are applicable. On this basis, for the purposes of the stability analyses of the proposed steepened beach, a residual strength ratio of 0.11 has been assumed.

4.3 Infinite slope stability assessment

The outputs of the consolidation analysis enabled a spreadsheet-based infinite slope analysis to be developed. This analysis was made across the range of depths relevant to the deposit at a given time increment.

FS was calculated using the infinite slope formula outlined as follows:

$$FS = \frac{s_r}{h\gamma \sin\alpha \cos\alpha} \tag{1}$$

Where:

- s_r = residual shear strength.
- $h\gamma$ = total stress at the potential failure surface.
- α = slope angle.

The results at 365 days of deposition at 5 m/year are outlined in Figure 6 for 1, 1.5, and 2% beach slopes. The Australian National Committee on Large Dams (ANCOLD) (2012) suggests a post-seismic FS requirement ranging from 1-1.2, depending on confidence in the selection of residual shear strength. The analyses outlined here made a number of conservative assumptions, including that the foundations soils were completely impermeable, and that the minimum residual strength would be developed across the entire deposit. On this basis, the minimum FS of 1.09 for the 1.5% beach slope case is deemed to be sufficient.

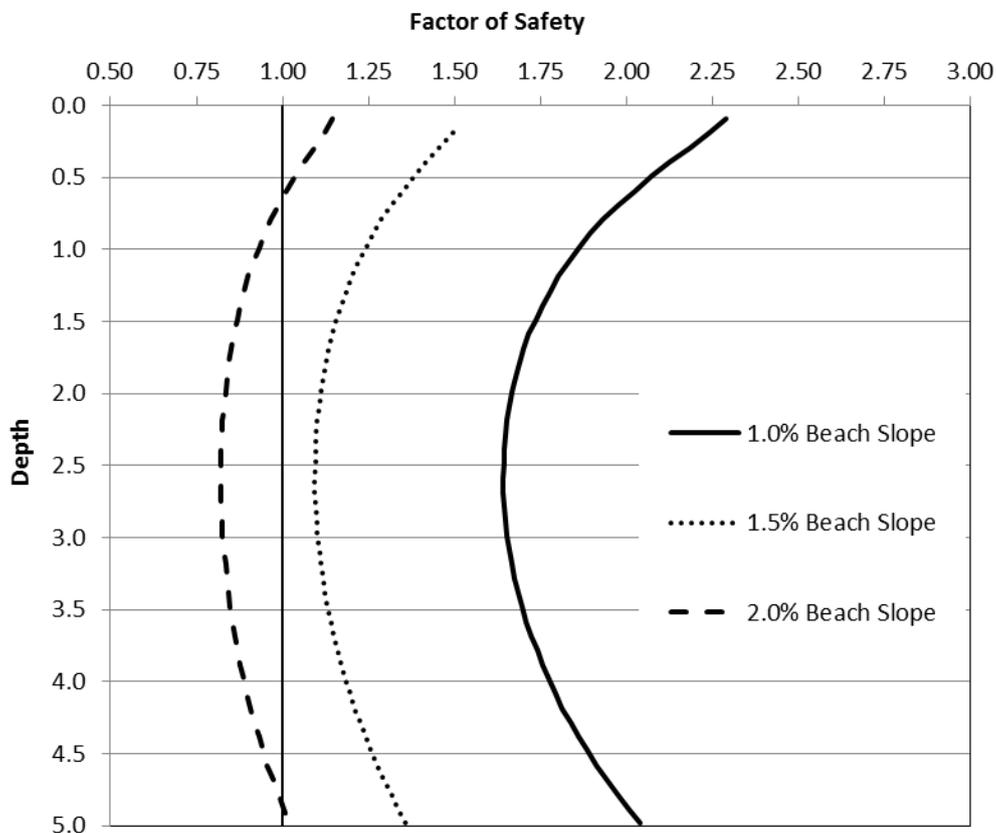


Figure 6 Infinite slope stability results – different beach slopes at 365 days deposition

The results for the 1.5% slope are presented in Figure 7 for time increments of 100, 200, and 365 days of deposition. This indicates that as deposition continues, the FS generally decreases. This is a result of the deposition rate at 5 m/year being faster than the tailings can consolidate. Therefore, as deposition continues, the relationships between in situ effective stress and driving forces become worse with respect to stability. Importantly, this suggests that even were some of the material properties to be slightly different to those observed in this study, monitoring of the tailings should enable this to be realised prior to an unacceptable FS developing.

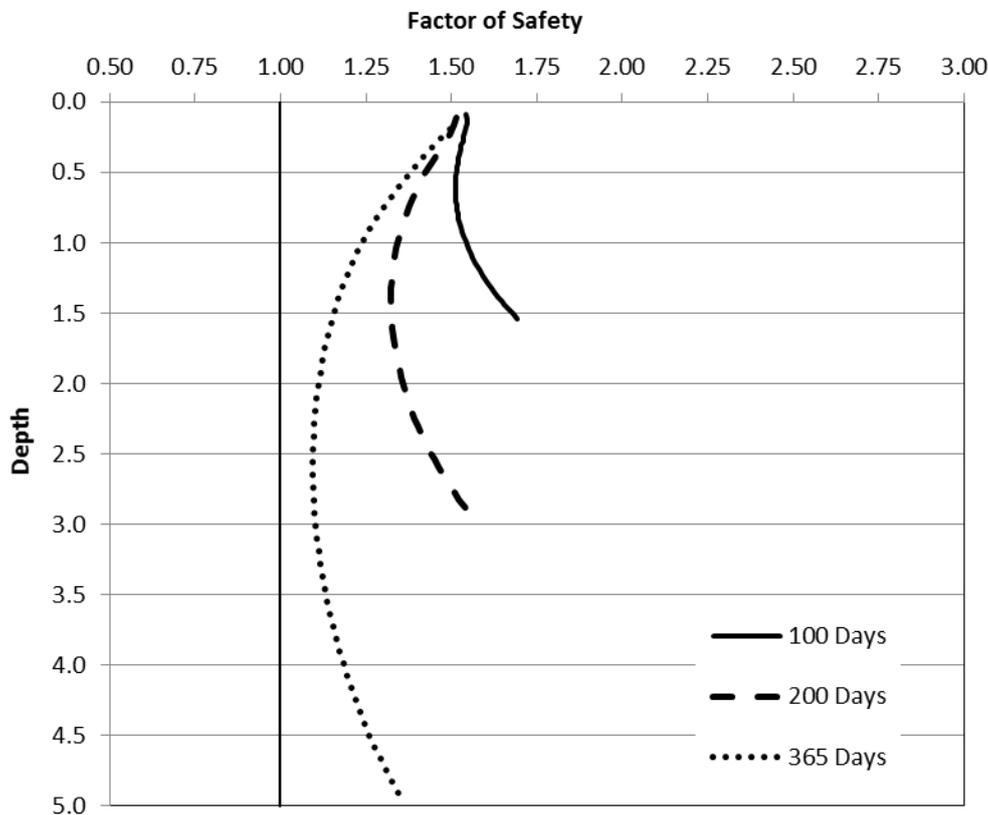


Figure 7 Infinite slope stability results – 1.5% beach slope

5 Conclusions

A laboratory test program was undertaken to characterise a lateritic tailings material following PT, as relevant to the assessment of consolidation and stability of a potential steepened beach concept. The results indicated that the material may undergo strength loss following sufficient seismic loading. On this basis, residual strengths were estimated using a laboratory-scale shear vane. This indicated residual strength ratios of 0.11, consistent for similar materials. Consolidation and settling tests were undertaken, and a consolidation model of the proposed deposition developed. The outputs of the consolidation model were utilised at 100, 200, and 365 days within an infinite-slope stability assessment. This indicated that assuming residual strengths were developed, the steepened beach concept was likely to be stable at beach slopes at or below 1.5%.

Limitations to the current study include the slight varying of polymer treatment techniques during the laboratory testing, applying the residual strength ratios obtained at 29-50 kPa to an analysis relevant to lower stresses, and the scarcity of permeability data across the relevant range of stresses. To address these limitations, additional testing is proposed on material deposited during the early stages of the trial, to confirm design assumptions. Further, as the results indicate that FS generally decreases with time at the ROR envisaged, confirmation testing in the early stages of deposition should indicate a potential unsatisfactory situation before it develops.

Acknowledgement

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